

6.0 Results of Risk Analyses

A risk analysis of the optimized embankment designs for the Salton Sea restoration project was conducted jointly by Kleinfelder and representatives from Reclamation. A risk evaluation team (RET) was convened from August 7th through August 11th, 2006 to evaluate the risk of various embankment failures under static and seismic loadings. The meeting was held at Reclamation's Technical Service Center (TSC) in Denver, Colorado. This chapter provides a brief summary of the methods and results of the risk analysis. Supporting information, failure mode descriptions, event factor evaluations and risk modeling results are presented in Appendix 2D.

6.1 Methodology

The risk analysis was performed in accordance with Reclamation's guidelines. The likelihood of various loadings, and the corresponding likelihood of structural responses were estimated for each of the embankment configuration options. In addition, the uncertainty surrounding each factor was quantified. The RET estimated risks in terms of Annual Probability of Failure and Annualized Loss of Life. These items are defined as follows:

$$\text{Annual Probability of Failure} = (\text{Probability of the Loading}) \times (\text{Probability of Failure given the Loading})$$

$$\text{Annualized Loss of Life} = (\text{Probability of the Loading}) \times (\text{Probability of Failure given the Loading}) \times (\text{Adverse Consequences given the Failure})$$

Where:

- < *Probability of the Loading* is the annual probability that the chosen load range responsible for a failure will occur.
- < *Probability of Failure given the Loading* is the likelihood that the dam will fail under the specific loading (ranges from 0 to 1.0).
- < *Adverse Consequences given the Failure* is typically expressed in terms of the estimated number of lives lost given a dam failure

The RET broke down each failure mode into a detailed "event tree" that includes the individual steps or components that sequentially lead to dam failure. Thorough discussions were held on the factors that affect each branch of the event tree, and then the risk team estimated the associated probabilities for those branches.

Individual estimates of probability for each branch were given a range of probability to reflect the team's judgment on the level of uncertainty. This range was expressed in the form of a function having a probable low, best estimate, and probable high. The computer program @RISK was utilized to compute the probability of failure and the annualized loss of life. The @RISK program uses a Monte Carlo analysis to simulate the range of distributions and results of each branch of the event tree, and to combine all branches to show the overall range of risk for a given failure mode. The resulting risk values were then plotted on graphs showing the comparison to Reclamation's Public Protection Guidelines (Reclamation, 2003).

6.2 Alternatives and Embankments Evaluated

The five alternatives described in the preceding sub-sections each require different embankments to achieve the desired water storage and management objectives. A summary of the required embankments and the design criteria for all but the "low hazard" dikes for the habitat ponds has been provided in Table 3.2 of subsection 3.7.

The embankments listed in Table 3.2 are described in detail in Chapters 4.0 and 5.0 and are shown on Figures 4.10, 4.12, 4.13, and 4.14, and 5.1 through 5.3.

6.3 Potential Failure Modes

The RET considered whether each embankment configuration option would have distinctly different failure modes. After review of the site conditions and proposed embankment configurations, the team concluded that some failure modes were likely to be common to all the proposed structures, with many similarities due to similar foundation geology and the selection of a common seismic design standard (yield acceleration equal to or greater than 0.17g, see Appendix 2B, Seepage and Stability Analyses for more details). Differences were generally attributable to differences in embankment configurations and/or detection, mitigation, or removal of problematic foundation geologic materials.

Table 6.1 summarizes the failure modes that were identified and evaluated during this risk analysis. Because of the similarity of failure modes for all of the alternative structures, the RET adopted an approach of evaluating a set of "common" failure modes using the "optimized" mid-Sea dam and south-Sea dam configurations for the base assessments. Then the team assessed how the other alternative conditions differed from those configurations, leading to either fewer or additional needed conditions for failure (i.e., branches) and/or the increased or decreased likelihood of each individual condition.

There are two important notes related to the potential failure modes summarized in Table 6.1. First, at the outset of this study, one of the fundamental assumptions was that liquefaction of materials within the upper stiff lacustrine deposit would

not occur. Consideration of liquefaction would be constrained to materials within the upper alluvial deposits. However, the RET, upon careful consideration of the available subsurface information, believes the morphology of the seafloor deposits is such that there is some likelihood that liquefiable (and erodible) layers and lenses exist within the soft lacustrine and upper stiff lacustrine deposits. As such, this possibility was considered and assumed in the risk analysis as failure mode (FM) No. 6. The “optimized” cross-sections evaluated as part of the risk analysis were developed to meet static and seismic design criteria for the potential of liquefaction within the upper alluvial and soft lacustrine deposits. Further refinement of the cross-sections would be required to meet seismic design criteria should future site explorations identify potentially liquefiable materials within the upper stiff lacustrine deposits. The results of the risk analysis concerning FM No. 6 confirm this requirement.

Second, the potential for fault offset that would translate through the seafloor deposits to the base of embankment structures crossing the Imperial / San Andreas faults and Fault Transition Zone (see Figure 1.2) was not a primary design consideration prior to the risk analysis. However, during the risk analysis meeting, Reclamation personnel identified the potential for surface expression of fault ruptures that could be as much as 2 to 5 meters. As such, the RET considered this as a potential failure mode (FM No. 12). Similar to the situation described above for the potential for liquefaction within the upper stiff lacustrine deposits, further refinement of the cross-section of embankments crossing this fault offset zone would be required to meet seepage design criteria and to reduce the potential for failure following a seismic event that would cause surface rupture of the seafloor deposits. For example, thicker zones within the various embankment cross sections might mitigate this problem. The results of the risk assessment confirm this requirement.

Table 6.1
Summary of Risk Estimates for All Embankment Structures

Component	Sub-component	Static - Internal Erosion (Piping) of Embankment	Static - Internal Erosion of Foundation Materials	Seismic - Deformation and Overtopping of Embankment	Seismic - Deformation and Internal Erosion of Embankment	Seismic - Deformation and Internal Erosion of Foundation Materials	Seismic - Liquefaction of Upper Stiff Lacustrine, Deformation and Overtopping of Embankment	Seismic - Offset and Translation of Embankment
Mid-Sea-Dam	Sand Dam with stone columns	FM1	FM2	FM3	FM4	FM5	FM6	
	Rockfill Dam with Rock Notches with Maximum Seismic Filters	FM7 ≤ FM1	FM8	FM9	FM10 ≤ FM4	FM11	FM6	
Mid-Sea-Barrier		≤ FM1 (no stone columns)	≤ FM2 (no stone columns)	≤ FM3 (with stone columns)	≤ FM4 (with stone columns)	≤ FM5 (with stone columns)		
Perimeter Dike		≤ FM1	= FM2	≤ FM3	≤ FM4	≤ FM5	≤ FM6	
South-Sea Dam		≤ FM1	≤ FM2	≤ FM3	≤ FM4	≤ FM5	≤ FM6	= FM12
North-Sea Dam		≤ FM1	≤ FM2	≤ FM3	≤ FM4	≤ FM5	≤ FM6	
Concentric Lakes Dikes		≤ FM1 ^(a) 1x10 ^{-2 (b)}	≤ FM2 ^(a) 1x10 ^{-2 (b)}	≤ FM3 ^(a) 1x10 ^{-2 (b)}	≤ FM4 ^(a) 1x10 ^{-3 (b)}	≤ FM5 ^(a)	≤ FM6 ^(a)	= FM12 _(a,b)
Habitat Pond Embankments		1.0x10 ^{-4 (c)}	1.0x10 ^{-4 (c)}	1.0x10 ^{-2(c)}	1.0x10 ^{-3(c)}			≤ FM12

Notes: a) These values are estimated for cross-section meeting seismic design criteria and require APF of 1.0E-04. This could include the outer 1 to 2 lakes.
b) These values are estimated for cross-section that does not meet seismic or seepage design criteria. This could be adopted for the inner lakes.
c) These values are based on expert opinion using levee performance data. No detailed evaluation performed for this estimate.

For each structure, the RET evaluated risks associated with static and seismic failure modes. No hydrologic failure modes were considered in this risk analysis. In previous studies (Reclamation, 2005), Reclamation had evaluated the possibility of hydrologic failure modes and determined that they were unlikely to impossible. Members of the current RET reviewed operational conditions for each of the restoration alternatives. Since the inflows for each of the alternatives will be highly controlled, the risk of a hydrologic loading condition that would lead to overtopping, spillway or outlet structure failures was again judged as unlikely to impossible. Given these factors, the RET concluded that there are no plausible hydrologic failure modes expected to pose any appreciable risk. Accordingly, no detailed evaluations of the hydrologic failure modes were performed for this risk analysis.

In general, the RET evaluated three categories of static failure modes: internal erosion of the embankment, internal erosion of the foundation, and internal erosion of the embankment into the foundation. Seismic failure modes included failure due to overtopping, seismic cracking (through seepage), seismic under seepage, liquefaction of the foundation, and failures due to fault displacement.

6.4 Estimation of Annual Probability of Failure and Loss of Life

As each of the failure modes was defined and then understood, the RET began the process of discussing each step of the failure mode and assessing annual probabilities of failure. For all of the above plausible failure modes, event trees were utilized to assess the overall probability of dam failure. These event trees are provided in Attachment B of Appendix 2D.

The team also developed estimates for the population-at-risk, flood severity, consequences and the associated potential for loss of life (LOL) for each of the failure modes. In general, access to the project structures will be closed to the public and there are no permanent residents downstream that could be exposed to dam failure flooding. The population-at-risk is primarily associated with recreation activities and maintenance and operation of the facilities. Accordingly, the population-at-risk and the potential loss of life for this project are very low as summarized in Table 9.1.

The team then developed LOL distributions to be used for calculations of the annualized loss of life (ALL) for each embankment structure. The annual probability of failure estimates were then multiplied by the estimated loss of life to calculate the annualized loss of life risks posed by the potential failure modes. The mean annual probability of failure and annualized loss of life estimates for static and seismic failure modes evaluated for each structure are summarized in Tables 6.2 through 6.9.

The APF (and ALL) values displayed for FM No. 6 in Tables 6.2 through 6.9 below assume that an adequate geotechnical investigation to detect liquefiable layers/lenses within the upper stiff lacustrine deposits would not be performed, and that the embankment designs would not be revised accordingly. Similarly, the APF (and ALL) values displayed in these tables for FM No. 12 assume that where the dam and/or dike alignments might be located in an area that could experience earthquake-induced fault offsets in their foundation (such as the south end of the Sea), the geologic exploration work to identify such locations and the design work to develop appropriate embankment designs (i.e., thicker zones) would not be done. An important intent of these two failure modes and their negative assumptions was to identify the potential consequences of a failure and to identify and mitigate these concerns. The RET believes that mitigation measures including appropriate design and construction level investigations and adaptive design for FM No. 6, and modification of the dam cross-section to withstand the large horizontal and vertical displacements that would occur in the general region where fault rupture is expected for FM No. 12 could be undertaken so that APF and ALL values would easily meet Reclamation's dam safety guidelines. Therefore, these APF (and ALL) values for FM Nos. 6 and 12 in Table 6.2 through 6.9 should be considered as a potential "worst case" condition and not valid regarding the actual risks for these dam and dike embankment designs.

Table 6.2
Summary of Mean Risk Estimates
Mid-Sea Dam Option A, Sand Dam with Stone Columns

Failure Mode	Failure Mode Description	Mean APF	LOL	Mean ALL
FM No. 1	Mid-Sea-Dam, Static - Internal Erosion (Piping) of Embankment	3.8E-11	0	0
FM No. 2	Mid-Sea-Dam, Static - Internal Erosion of Foundation Materials	6.1E-09	0	0
FM No. 3	Mid-Sea-Dam, Seismic - Deformation and Overtopping of Embankment	3.8E-06	2	7.6E-06
FM No. 4	Mid-Sea-Dam, Seismic - Deformation and Internal Erosion of Embankment	3.1E-11	2	6.2E-11
FM No. 5	Mid-Sea-Dam, Seismic - Deformation and Internal Erosion of Foundation Materials	8.0E-08	2	1.6E-07
FM No. 6	Mid-Sea-Dam, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	4.0E-05	2	8.0E-05
Overall Risk (maximum risk for static and seismic FMs)		4.0E-05	2	8.0E-05

Note: LOL values are based on best estimate values from Table 2D.29 in Appendix 2D. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0.

Table 6.3
Summary of Mean Risk Estimates
Mid-Sea Dam Option D, Rockfill Dam with Rock Notches, Maximum
Seismic Filters

Failure Mode	Failure Mode Description	Mean APF	LOL ^(a)	Mean ALL
FM No. 7	Rock Notches, Static - Internal Erosion of Embankment	$\leq 3.1\text{E-}11$	0	0
FM No. 8	Rock Notches, Static - Internal Erosion of Foundation Materials	$2.3\text{E-}07^{(b)}$	0	0
FM No. 9	Rock Notches, Seismic - Deformation and Overtopping of Embankment	$1.0\text{E-}15$	2	$2.0\text{E-}15$
FM No. 10	Rock Notches, Seismic - Deformation and Internal Erosion of Embankment	$\leq 3.1\text{E-}11$	2	$\leq 6.2\text{E-}11$
FM No. 11	Rock Notches, Seismic - Deformation and Internal Erosion of Foundation Materials	$3.1\text{E-}07^{(b)}$	2	$6.2\text{E-}07$
FM No. 6	Rock Notches, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	$4.0\text{E-}05$	2	$8.0\text{E-}05$
Overall Risk (maximum risk for static and seismic FMs)		$4.0\text{E-}05$	2	$8.0\text{E-}05$

Note: a) LOL values are based on best estimate values from Table 2D.29 in Appendix 2D.

Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0.

b) These values derived through the Risk Analysis are in the range where there is decreasing justification to take action to reduce risk in the long or short term. However, the design configurations do not meet Reclamation design criteria for “full” filters.

Table 6.4
Summary of Mean Risk Estimates
Mid-Sea Barrier

Based on Failure Mode	Failure Mode Description	Mean APF	LOL	Mean ALL
FM No. 3 (without stone columns)	Mid-Sea-Barrier, Seismic - Deformation and Overtopping of Embankment	$>1.0E-02$	0	0
FM No. 3 (with stone columns)	Mid-Sea-Barrier, Seismic - Deformation and Overtopping of Embankment	$\leq 3.8E-06$	0	0
Overall Risk (maximum risk for static and seismic FMs) Without Stone Columns		$>1.0E-02$	0	0
Overall Risk (maximum risk for static and seismic FMs) With Stone Columns		$\leq 3.8E-06$	0	0

Note: LOL values are based on best estimate values from Table 2D.29 in Appendix 2D. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0.

Table 6.5
Summary of Mean Risk Estimates
Perimeter Dikes (with Stone Columns)

Based on Failure Mode	Failure Mode Description	Mean APF	LOL	Mean ALL
FM No. 1	Perimeter Dikes, Static - Internal Erosion (Piping) of Embankment	$\leq 3.8\text{E-}11$	0	0
FM No. 2	Perimeter Dikes, Static - Internal Erosion of Foundation Materials	$6.1\text{E-}09$	0	0
FM No. 3	Perimeter Dikes, Seismic - Deformation and Overtopping of Embankment	$\leq 3.8\text{E-}06$	0	0
FM No. 4	Perimeter Dikes, Seismic - Deformation and Internal Erosion of Embankment	$\leq 3.1\text{E-}11$	0	0
FM No. 5	Perimeter Dikes, Seismic - Deformation and Internal Erosion of Foundation Materials	$\leq 8.0\text{E-}08$	0	0
FM No. 6	Perimeter Dikes, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	$\leq 4.0\text{E-}05$	0	0
Overall Risk (maximum risk for static and seismic FMs)		$\leq 4.0\text{E-}05$	0	0

Note: LOL values are based on best estimate values from Table 2D.29 in Appendix 2D. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0.

Table 6.6
Summary of Mean Risk Estimates
South-Sea Dam (with Stone Columns)

Based on Failure Mode	Failure Mode Description	Mean APF	LOL ^(a)	Mean ALL
FM No. 1	South-Sea Dam, Static - Internal Erosion (Piping) of Embankment	$\leq 3.8\text{E-}11$	0	0
FM No. 2	South-Sea Dam, Static - Internal Erosion of Foundation Materials	$\leq 6.1\text{E-}09$	0	0
FM No. 3	South-Sea Dam, Seismic - Deformation and Overtopping of Embankment	$\leq 3.8\text{E-}06$	1	$\leq 3.8\text{E-}06$
FM No. 4	South-Sea Dam, Seismic - Deformation and Internal Erosion of Embankment	$\leq 3.1\text{E-}11$	1	$\leq 3.1\text{E-}11$
FM No. 5	South-Sea Dam, Seismic - Deformation and Internal Erosion of Foundation Materials	$\leq 8.0\text{E-}08$	1	$\leq 8.0\text{E-}08$
FM No. 6	South-Sea Dam, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	$\leq 4.0\text{E-}05$	1	$\leq 4.0\text{E-}05$
FM No. 12	South-Sea Dam, Seismic - Offset and Translation of Embankment (with translation mitigation design features)	$1.0\text{E-}04$	1	$1.0\text{E-}04$
Overall Risk (maximum risk for static and seismic FMs)^(b)		$1.0\text{E-}04$	1	$1.0\text{E-}04$

Notes: a) LOL values are based on best estimate values from Table 2D.29 in Appendix 2D. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0.

b) Maximum overall risk would be $1.1\text{E-}02$ and annualized loss of life would be $1.1\text{E-}02$ if translation mitigation design features are not incorporated in the design.

Table 6.7
Summary of Mean Risk Estimates
North-Sea Dam (with Stone Columns)

Based on Failure Mode	Failure Mode Description	Mean APF	LOL	Mean ALL
FM No. 1	North-Sea Dam, Static – Internal Erosion (Piping) of Embankment	$\leq 3.8\text{E-}11$	0	0
FM No. 2	North-Sea Dam, Static – Internal Erosion of Foundation Materials	$\leq 6.1\text{E-}09$	0	0
FM No. 3	North-Sea Dam, Seismic - Deformation and Overtopping of Embankment	$\leq 3.8\text{E-}06$	2	$\leq 7.6\text{E-}06$
FM No. 4	North-Sea Dam, Seismic - Deformation and Internal Erosion of Embankment	$\leq 3.1\text{E-}11$	2	$\leq 6.2\text{E-}11$
FM No. 5	North-Sea Dam, Seismic - Deformation and Internal Erosion of Foundation Materials	$\leq 8.0\text{E-}08$	2	$\leq 1.6\text{E-}07$
FM No. 6	North-Sea Dam, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	$\leq 4.0\text{E-}05$	2	$\leq 8.0\text{E-}05$
Overall Risk (maximum risk for static and seismic FMs)		$\leq 4.0\text{E-}05$	2	$\leq 8.0\text{E-}05$

Note: LOL values are based on best estimate values from Table 2D.29 in Appendix 2D. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0.

Table 6.8
Summary of Mean Risk Estimates
Concentric Lakes Dikes

Based on Failure Mode	Failure Mode Description	Mean APF	LOL ^(d)	Mean ALL
FM No. 1	Lakes Dikes, Static - Internal Erosion (Piping) of Embankment	$\leq 3.8\text{E-}11^{(a)}$ $1.0\text{E-}02^{(b)}$	0	0
FM No. 2	Lakes Dikes, Static - Internal Erosion of Foundation Materials	$\leq 6.1\text{E-}09^{(a)}$ $1.0\text{E-}02^{(b)}$	0	0
FM No. 3	Lakes Dikes, Seismic - Deformation and Overtopping of Embankment	$\leq 3.8\text{E-}06^{(a)}$ $1.0\text{E-}02^{(b)}$	0	0
FM No. 4	Lakes Dikes, Seismic - Deformation and Internal Erosion of Embankment	$\leq 3.1\text{E-}11^{(a)}$ $1.0\text{E-}03^{(b)}$	0	0
FM No. 5	Lakes Dikes, Seismic - Deformation and Internal Erosion of Foundation Materials	$\leq 8.0\text{E-}08^{(a)}$	0	0
FM No. 6	Lakes Dikes, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	$\leq 4.0\text{E-}05^{(a)}$ $\leq 8.4\text{E-}03^{(b)}$	0	0
FM No. 12	South-Sea Dam, Seismic - Offset and Translation of Embankment (with translation mitigation design features)	$1.0\text{E-}04^{(c)}$	0	0
FM No. 12	South-Sea Dam, Seismic - Offset and Translation of Embankment (without translation mitigation design features)	$1.1\text{E-}02^{(c)}$	0	0
Overall Risk (maximum risk for static and seismic FMs)		$1.0\text{E-}02^{(c)}$	0	0

Notes: a) These values are estimated for an improved cross-section meeting seismic design criterion. For example, the outer 1 to 2 lakes would be designed to meet the seismic design criteria.

b) These values are estimated for unimproved cross-sections that do not meet seismic or seepage design criteria. This could be adopted for the remaining inner lakes.

c) Maximum overall risk of failure would be $1.1\text{E-}02$ but annualized loss of life would be zero if translation mitigation design features are not incorporated in the design for "outer" lakes.

d) LOL values are based on best estimate values from Table 2D.29 in Appendix 2D. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0.

Table 6.9
Summary of Mean Risk Estimates
Habitat Pond Embankments

Based on Failure Mode	Failure Mode Description	Mean APF	LOL^(b)	Mean ALL
N/A ^(a)	Habitat Ponds, Static - Internal Erosion (Piping) of Embankment	1.0E-04	0	0
N/A ^(a)	Habitat Ponds, Static - Internal Erosion of Foundation Materials	1.0E-04	0	0
N/A ^(a)	Habitat Ponds, Seismic - Deformation and Overtopping of Embankment	1.0E-02	0	0
N/A ^(a)	Habitat Ponds, Seismic - Deformation and Internal Erosion of Embankment	1.0E-03	0	0
N/A ^(a)	Habitat Ponds, Seismic - Deformation and Internal Erosion of Foundation Materials	≤ 1.1E-02	0	0
N/A ^(a)	Habitat Ponds, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	≤ 1.1E-02	0	0
FM No. 12	Habitat Ponds, Seismic - Offset and Translation of Embankment	≤ 1.1E-02	0	0
Overall Risk (maximum risk for static and seismic FMs)		1.0E-02	0	0

Notes: a) These values are based on expert opinion using levee performance data. No detailed evaluation performed for this estimate.

b) LOL values are based on best estimate values from Table 2D.29 in Appendix 2D. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0.

6.5 Compilation of Highest Annual Probability of Failure and Loss-of-Life for Each Restoration Alternative

After the RET had evaluated the risks for all of the failure modes for each structure, the team then compiled the risks for each structure to develop a “composite” risk for each alternative. The group considered that since all static and seismic loadings for each individual structure were not independent variables (i.e., each structure would experience the loading for the same particular event at the same time, rather than as separate and independent events), the risk of failure of an alternative could be described by the risk associated with failure of the

“weakest link” in the system. Therefore, Annual Probability of Failure and Annualized Loss of Life for each alternative was considered to be the highest value for static or seismic failures for each of the structures comprising an alternative.

While the RET considered the potential for liquefaction within the upper stiff lacustrine deposit (FM No. 6) and translation failure due to fault rupture (FM No. 12), as noted above, the RET believes that these risks can be mitigated. When the potential for mitigation of liquefaction concerns within the upper stiff lacustrine, or translation due to fault rupture is considered, risks for each structure reduces significantly. Table 6.10 presents a summary of the estimated risks with mitigation of potential liquefaction within the upper stiff lacustrine deposit or fault translation concerns assumed to have occurred. Additional discussion related to these risks and associated risk mitigation is provided in Appendix 2D. Under these conditions, the risks for all alternatives would meet Reclamation guidelines.

Table 6.10
Summary of Alternative Risks

Embankment	Mid-Sea Dam/North Marine Lake (1) Salton Sea Authority Alternative			Mid-Sea Barrier/South Marine Lake (with stone columns) (2)			Concentric Lakes Dikes (3)			North-Sea Dam/Marine Lake (4)			Habitat Pond Embankments (5)		
Mid-Sea Dam (Sand Dam with stone columns)	APF 3.8 E-06	LOL 2	ALL 7.6 E-06												
Mid-Sea Barrier				APF ≤ 3.8 E-06	LOL 0	ALL 0									
Perimeter Dikes	APF ≤ 3.8 E-06	LOL 0	ALL 0												
South-Sea Dam	APF 1.0 E-04	LOL 1	ALL 1.0 E-04												
North-Sea Dam										APF ≤ 3.8 E-06	LOL 2	ALL ≤ 7.6 E-06			
Concentric Lakes Dikes (with translation mitigation design features)							APF 1.0 E-04	LOL 0	ALL 0						
Habitat Pond Embankments													APF 1.0 E-02	0	ALL 0
Controlling Maximums	APF 1.0 E-04	LOL 2	ALL 1.0 E-04	APF ≤ 3.8 E-06	LOL 0	ALL 0	APF 1.0 E-04	LOL 0	ALL 0	APF ≤ 3.8 E-06	LOL 2	ALL 7.6 E-06	APF 1.0 E-02	LOL 0	ALL 0

6.6 Adaptations to Design Concepts Resulting from Initial Risk Analysis Results

The risk team discussed several potential revisions to the design concepts based on the initial risk analysis results. A summary of these recommendations is provided below. Additional information on these recommendations is provided in Appendix 2D.

- ✓ The SCB slurry wall should be extended to a depth of 40 feet into the upper stiff lacustrine deposit.
- ✓ The crest of the dam should be armored and reinforcing should be considered in the top of the SCB slurry wall.
- ✓ A blanket drain should be extended into the Type A material to provide for controlled collection and discharge of seepage through the SCB slurry wall following an earthquake event.
- ✓ The exploration programs for the design phase and during construction should be extensive and extend to a substantial depth into and below the upper stiff lacustrine deposit.
- ✓ The barrier and habitat pond embankment concepts may require cutoff walls in order to achieve the water control (balance) objectives of the project.
- ✓ The strains predicted by the FLAC model along the centerline of the dam (SCB wall location) show the maximum shears occurring at the contact of the dam to the stiff lacustrine material. The model, without considering different material properties associated with the SCB wall, estimates strains of up to 15% or about 0.75 foot over the 5-foot width of the element in the model. Such strains, although large, are tolerable for a plastic (HDPE) membrane that could be installed within the SCB wall. Consequently, the analysis results suggest that a membrane in the SCB wall could offer some important redundancy and protection for large seismic events.
- ✓ The risk analysis indicated that the membrane may offer 3 to 4 orders of magnitude of reduction of the probability of failure for the seismically induced seepage failure modes.
- ✓ Thickness of internal zones in south-Sea dam sand dam with stone columns concept could be increased to reduce the risk of failure due to translation (fault offset). Likewise, an internal blanket of coarser material in the Type A zone may also mitigate risks to some degree.
- ✓ Segmentation of the Salton Sea Authority alternative, such as by placing cross barriers connecting the west shore to the perimeter dike, would be prudent to mitigate the consequences of failure of the south-Sea dam or perimeter dike elements due to translation (fault offset).

6.7 Discussion of Risk and Cost of Replacement

The following paragraphs describe the anticipated risks and associated extent of repairs and replacement that may be required to each of the embankment components of the various restoration alternatives following a significant seismic event. For purposes of the discussions below, a significant earthquake is judged to have a recurrence interval between once every 100, to once every 200 to 250 years.

Mid-Sea, North- and South Sea Dams – Outer shells of all the preferred configurations of these dams (Sand Dams with Stone Columns) will be loose and subject to liquefaction and relatively large displacements during significant earthquake events. Riprap slope protection materials on the outer surface of the shells, particularly the portions of the shells that are saturated, will likewise be subject to relatively large deformations during and immediately following such a seismic event. It will be necessary to repair damage to the outer slopes including replacement of any displaced riprap in order to protect the dams from erosion from wave action. For the purpose of evaluating this risk and associated replacement and/or repair costs, it is estimated that between 30 and 50% of the lakeside outer slopes will require substantial repairs to the slopes and replacement of riprap slope protection materials. These dams have been designed to limit crest deformations (at the top of the stone column densified central section) to less than or equal to 5 feet. Therefore, it is likely that 2 to 4 feet of material will also need to be placed on the crest of the dam to restore the full freeboard following the significant earthquake event.

Mid-Sea Barrier – Two barrier cross-sections have been developed including a sand dam without stone columns meeting only static design criteria, and a sand dam with stone columns meeting both static and seismic design criteria. Similar to the mid-Sea dam as described above, the outer shells of these different cross-sections, including riprap slope protection materials, will be loose and subject to liquefaction and associated displacement during and immediately following a significant seismic event. For the cross-section meeting static only design criteria, not only will the outer shells deform, but the central section of the barrier will also displace along with the central seepage cutoff wall. For this cross-section it is likely that 50% or more of the entire dam will require complete replacement including the seepage cutoff wall. The remainder of the barrier will likely require substantial repairs and replacement of the outer slopes, replacement of riprap slope protection materials, localized repairs to the cutoff wall, and placement of materials on the crest of the dam to restore freeboard.

For the cross-section meeting both static and seismic design criteria it is estimated that between 30 and 50% of both outer shells will require substantial repairs to the slopes and replacement of the riprap slope protection materials. However, the central portion of the dam including the seepage cutoff wall will remain intact and

fully functional. Minor crest deformations will require the replacement of 2 to 4 feet of material to restore full freeboard.

Perimeter and Concentric Lakes Dikes – Similar to the mid-Sea barrier described above, alternative cross-sections meeting static only, or both static and seismic design criteria have been developed. The behavior of each of these alternatives is expected to be similar to that described for the mid-Sea barrier. Based on the limited understanding of the seafloor deposits around the Sea, it is estimated that between 30 and 50% of the dikes would require complete replacement following a significant earthquake event for the cross-section designed to meet static only design criteria. Likewise, 30 to 50% of the lakeside outer slopes and riprap material will require repair and/or replacement for the dikes meeting both static and seismic design criteria. In all cases, 1 to 2 feet of material will likely be required on the crest of the dikes to restore full freeboard protection.

Habitat Pond Embankments – The cross-section for these embankments have been designed to meet static design criteria only at this time. It is estimated that between 30 and 40% of these embankments will require substantial repairs or replacement following a significant earthquake event in order to restore the full function of the habitat pond complexes.